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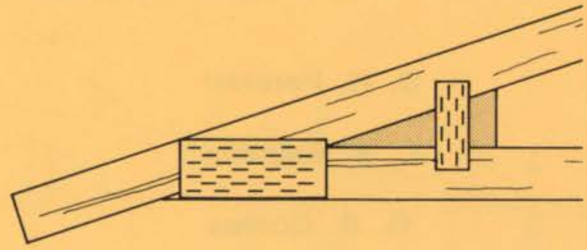

TECHNICAL NOTE NO. 12

Small Homes Council–Building Research Council, University of Illinois at Urbana-Champaign

INFLUENCE OF HEEL WEDGES ON THE STIFFNESS AND STRENGTH OF WOOD ROOF TRUSSES WITH METAL PLATE CONNECTIONS

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Twenty-five Cents

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Abstract

Twenty-four experimental metal-plate wood roof trusses were tested on four truss types with six specimens in each category. The object of the study was to determine the effect of adding wood wedges from scrap cutoff material to the heel joint. The influence of wedges on both the deflection behavior and the strength of the trusses was observed and analyzed. The King-Post truss was significantly influenced by the addition of the wedge. The deflection behavior of the Fink and Howe type trusses was improved, but only by a small amount. The strength of the Fink and Howe trusses was judged to be unaffected by the addition of wedges.

UTILIZATION OF THE WEDGE SHAPED PIECE trimmed from the miter cut of the lower chord of common types of wood trusses would solve a disposal problem and could possibly increase the efficiency of the product. This investigation was concerned with the use of the wedge in reinforcing the heel joint (Fig. 1). Two questions were posed: one was concerned with the influence of the wedge on deflection characteristics of the truss, and the other was concerned with possible improvements in strength.

The study was designed to yield a very complete set of data on each truss and also to replicate the individual tests in order to provide estimates of the variation that can be expected in their performance. Four basic truss types were studied: 1) 30 foot-0 inch span, 2/12 slope, Fink pattern; 2) 29 foot-8 inch span, 2/12 slope, Howe pattern; 3) 32 foot-4 inch span, 4/12 slope, Howe pattern; and 4) 19 foot-8 inch span, 4/12 slope, King Post pattern. These trusses were experimentally planned to exceed the maximum spans permitted by the Truss Plate Institute Design Specification.[1] Maximum spans according to the latter specification are 28 foot-6 inches, 28 foot-4 inches, 31

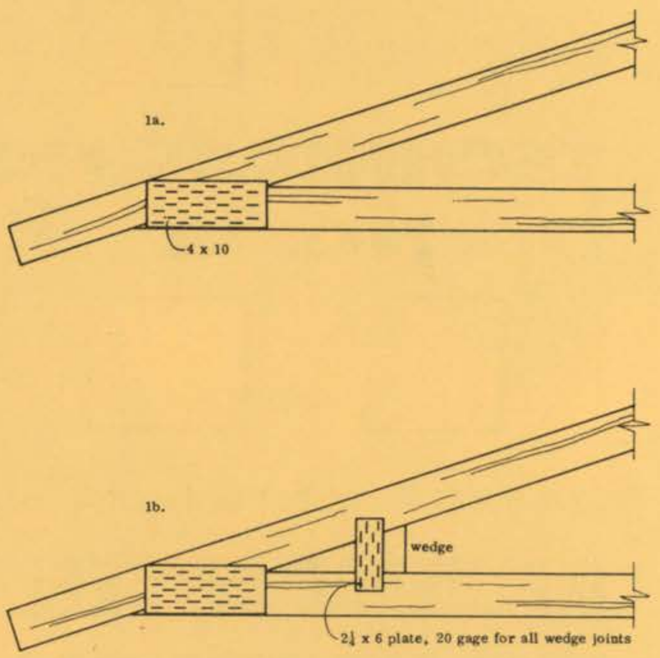

Figure 1. — A wedged heel joint (1b) is made from the common joint (1a) by adding the wood piece trimmed from the lower chord miter. The wedge becomes a part of the structural connection through the addition of paired plates that bind it between the upper and lower chords.

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[1]Truss Plate Institute. 1974. Design specifications for light metal plate connected wood trusses. TPI-74. College Park, Md.

foot-0 inches, and 18 foot-1 inch, respectively. Six trusses were fabricated in each of the 4 groups making a total of 24 trusses.

Deflection data were obtained at the design live load on each truss built without wedges followed by identical deflection tests with the wedges installed. Thus 24 paired sets of deflection data were obtained.

Following the deflection tests, half of the trusses in each group were loaded to failure with the wedges still in place. The wedges were removed from the remaining trusses, and these were also loaded to failure. Thus four groups of three trusses with wedges can be compared in strength performance with an equal number of counterparts without wedges.

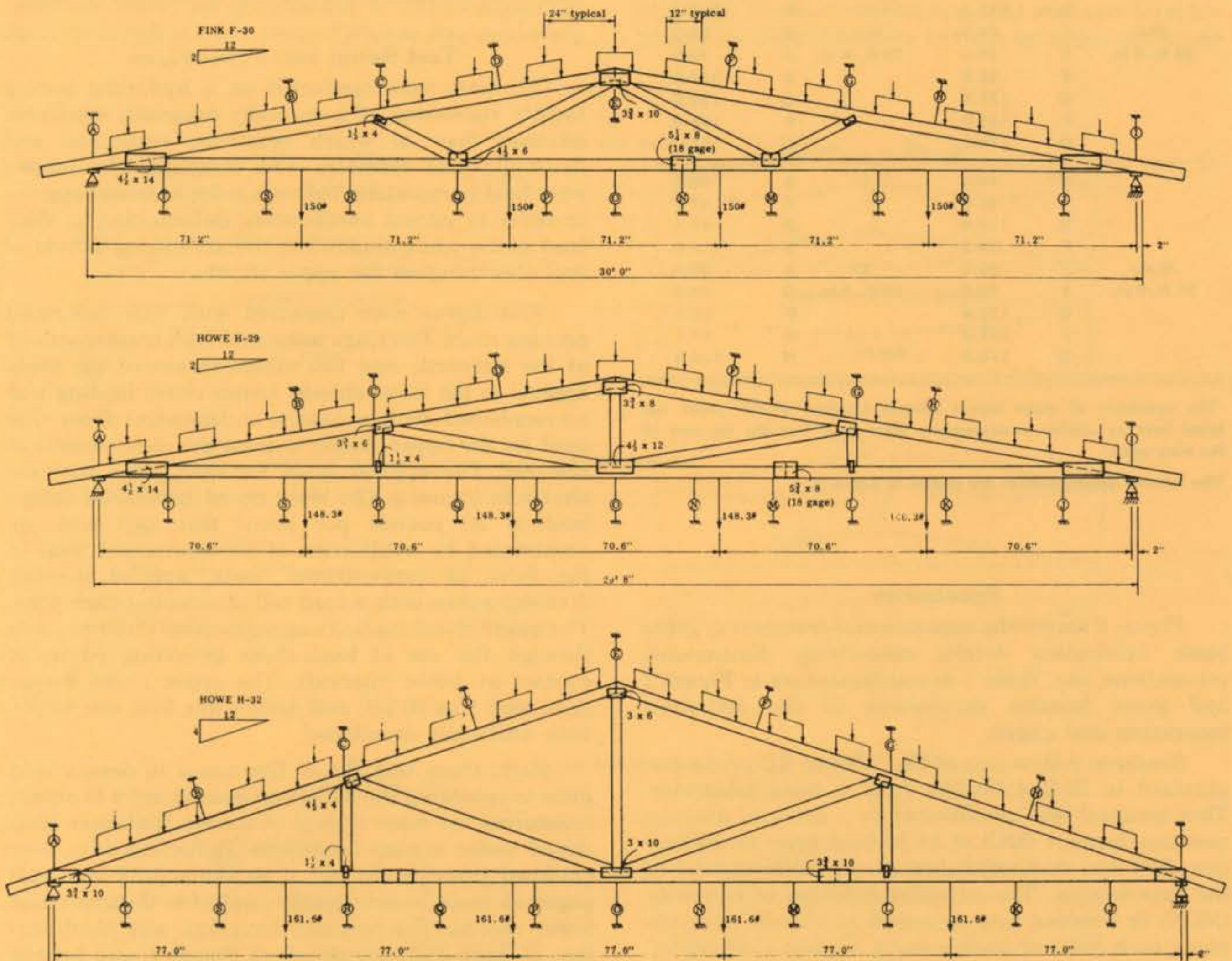

Figure 2. — The test truss types are shown under equivalent loads to produce design uniform dead load of 20 plf on each of the upper and lower chords plus design uniform live load of 60 plf on the upper chords. The 0.001-inch dial gage locations are shown with circular symbols containing the letter-code identification of each. All joint plates are 20 gage unless otherwise designated. The lumber is No. 1, Dense, K. D. grade, southern yellow pine. Dial gage locations, which are symmetrical about the truss center, are given in Table 1.

Table 1. — HORIZONTAL DISTANCES TO THE POINT OF CONTACT OF THE LISTED DIAL GAGE MEASURED FROM THE VERTICAL OF GAGE A.[1]

Truss	Gage[2]	Horizontal dist. from gage A (in.)	Truss	Gage[2]	Horizontal dist. from gage A (in.)
	B	69.0		B	50.5
	C	98.5		C	100.0
	D	139.9		D	145.5
	E	178.0		E	192.0
Fink	T	24.2	Howe	T	24.2
30 ft.-0 in.	S	59.0	32 ft.-4 in.	S	56.2
	R	88.5		R	100.0
	Q	125.0		Q	132.0
	P	155.0		P	164.0
	O	178.0		O	192.0
	B	69.0		B	20.5
	C	96.0		C	47.7
	D	137.4		D	89.4
	T	24.2		E	116.0
Howe	S	64.0	KP	R	20.5
29 ft.-8 in.	R	96.0	19 ft.-8 in.	Q	44.5
	Q	122.8		P	68.5
	P	149.0		O	92.5
	O	176.0		N	116.0

[1]The symmetry of gage layout permits location of the gages not listed here by similar measurements from gage I at the far end of the truss span.

[2]The lettered gage positions are shown in Figure 2.

Specimens

Figure 2 shows the experimental trusses and gives basic fabrication details concerning dimensions, connections, etc. Table 1 is supplementary to Figure 2 and gives location dimensions of the deflection measuring dial gages.

Southern yellow pine of No. 1 Dense KD grade was obtained in 20-foot lengths from a truss fabricator. This material was conditioned to a uniform average moisture content (MC) of 11 percent prior to fabrication. MC was monitored throughout the remainder of the experiments. The edgewise modulus of elasticity (MOE) in bending was measured at 18-inch intervals along each piece of lumber using a 3-foot-span center-point dead-load tester. MC at the span center of each MOE test was measured and these data recorded at the same time. These data, while not reported here, were taken to answer potential needs in later analytical work.

The metal gusset plates used were typical of contemporary types having punched teeth (frequently called nails) approximately 1/2 inch long and 20 gage in thickness. In some special joints, similar 18-gage plates were used as noted in Figure 2. Plate sizes and placement are also given in the same illustration. The wedges consisted of the angled cutoff portion from the lower chord and were held in place by a 20-gage plate as shown in Figure 1. The wedges were fitted loosely with the chords during placement in order to simulate the loss of mechanical contact that could result from shrinkage in actual practice.

The lumber parts were partially precut in the laboratory and transported to a commercial fabricator where the final cutting and other steps of manufacture were completed. This process was completed in 2-1/2 days, and the trusses were returned immediately to the laboratory environment in order to minimize unwanted delays in restoring their moisture equilibrium. Some problems arose in connection with the fabrication which affected the outcome of the experiments. These are discussed later in the section on results.

Test Setup and Procedures

All tests were conducted on a hydraulic testing facility consisting of a specially designed, reinforced concrete floor to which reactions, restraints, and hydraulic load cylinders were attached. The trusses were held horizontally between roller bearing supports in order to permit unrestrained deformation in their load plane while simulating the stabilizing effects of roof sheathing on the upper chords.

Test forces were measured with four calibrated proving rings; two rings measured total truss reactions at the supports; and two others measured the loads applied to the lower chords. Lower chord loading was accomplished with a system independent from that used for the upper chords to meet the requirements of the test. The applied loads for each truss type are shown in Figure 2. The lower chord distributed design load of 20 pounds per lineal foot (plf) was approximated by application of the equivalent total in the form of concentrated loads, applied through dividing yokes with a load cell monitoring each yoke. The upper chord loadings approximated uniform loads through the use of load shoes providing points of contact at 1-foot intervals. The upper chord design dead load was 20 plf, and design live load was 60 plf, both uniformly distributed.

Each truss was flexed five times to design load prior to recording the deflection data in order to obtain measurements more typical of trusses that have been flexed under service conditions. Deflection data were obtained through the use of as many 0.001-inch dial gages as could be practicably applied to the upper and lower chords. The location, direction, and identification of these dials are shown in Figure 2, and further details are given in Table 1. Those gages which were located directly above the reactions were used to correct all other readings for support settlement. The intent of the extensive gaging was to obtain as complete a picture as possible of the total deformation pattern of the test structure. The loading sequence used for all deflection tests consisted of the application of dead loads to both upper and lower chords followed by the application of live loads to the upper chord in quartered increments until the full live load level was reached. The high volume of data generated was later fed into permanent computer files which were then used to plot sequential deformation diagrams at each increment of load. The smooth transition of one load level into another and general symmetry of the plots were used to check the consistency of the individual readings.

Each truss was subjected to deflection test without wedges; then after the wedges were installed with a laboratory-built screw press, each truss was again flexed five times, and deflection data were recorded for the modified structure.

Each set of six trusses of the same type was divided into two groups of three. One group was tested to failure with the wedges in place while the other had the wedges removed to restore them to their original condition before being subjected to ultimate loading. Since the original numbering of trusses was random in nature, it was not considered necessary to exercise particular care in making these group separations. Dead load was applied to upper and lower chords followed by increased loading on the upper chord until failure occurred. Failure load levels were reached in 5 to 10 minutes.

Results

Figure 3 shows in graphical form all of the design load deflection data. Experimental deflection readings at design live load relative to a dead load zero level are plotted as individual dots to scale on upper and lower

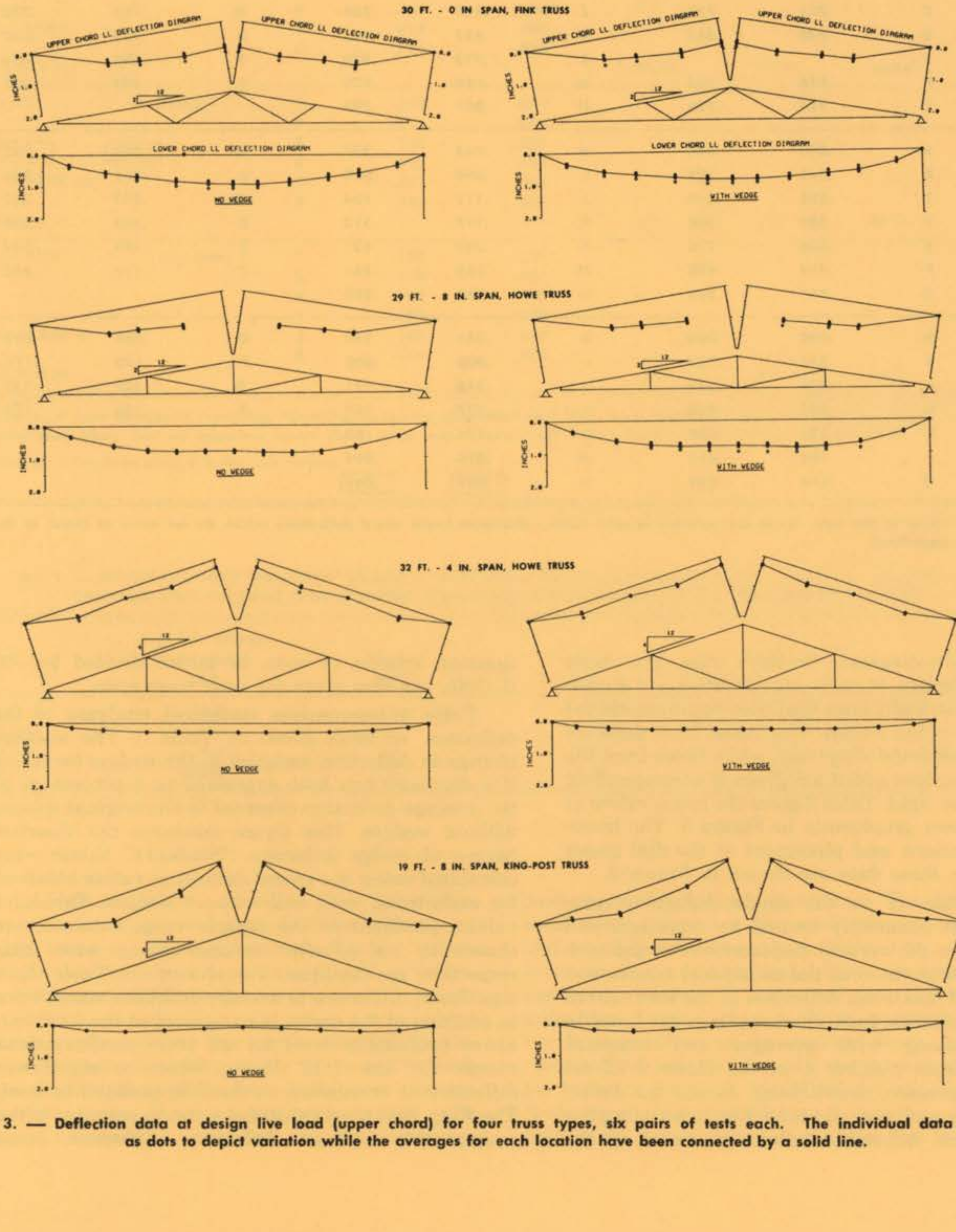

Figure 3. — Deflection data at design live load (upper chord) for four truss types, six pairs of tests each. The individual data are plotted as dots to depict variation while the averages for each location have been connected by a solid line.

Table 2. — AVERAGE DEFLECTIONS IN INCHES (CORRECTED FOR SINKING SUPPORTS) ARE SHOWN AT DESIGN LOAD FOR TRUSSES WITH AND WITHOUT WEDGES AT THE LETTERED GAGE LOCATIONS.[1]

Truss	Gage	No wedge	Wedge	Gage	No wedge	Wedge	Gage	No wedge	Wedge
	A	0.000	0.000	H	0.787	0.714	O	$\boxed{0.898}$	$\boxed{0.855}$
	B	.686	.665	I	.000	.000	P	.886	.848
Fink	C	.791	.764	J	.318	.351	Q	.818	.792
30 ft.-0 in.	D	.879	.876	K	.555	.609	R	.681	.692
	E	.807	.793	L	.685	.701	S	.559	.596
	F	.894	.873	M	.830	.803	T	.333	.344
	G	.786	.759	N	.899	.854			
	A	.000	.000	H	.746	.723	O	$\boxed{.757}$	$\boxed{.776}$
	B	.693	.668	I	.000	.000	P	.756	.769
Howe	C	.725	.746	J	.348	.360	Q	.765	.770
29 ft.-8 in.	D	.846	.869	K	.651	.693	R	.733	.737
	E			L	.795	.800	S	.596	.616
	F	.886	.905	M	.824	.829	T	.309	.321
	G	.787	.793	N	.801	.793			
	A	.000	.000	H	.344	.307	O	$\boxed{.263}$	$\boxed{.252}$
	B	.342	.303	I	.000	.000	P	.261	.248
Howe	C	.252	.240	J	.111	.124	Q	.269	.253
32 ft.-4 in.	D	.388	.390	K	.197	.212	R	.262	.250
	E	.236	.230	L	.258	.251	S	.194	.204
	F	.414	.400	M	.269	.254	T	.120	.135
	G	.247	.240	N	.262	.250			
	A	.000	.000	H	.351	.248	O	.086	.095
	B	.351	.224	I	.000	.000	P	.150	.178
King Post	C	.733	.529	J	.148	.191	Q	$\underline{.189}$	$\underline{.232}$
19 ft.-8 in.	D	.521	.392	K	$\underline{.177}$	$\underline{.242}$	R	.156	.183
	E	.051	.035	L	.134	.183			
	F	.488	.413	M	.076	.094			
	G	.694	.549	N	$\boxed{.047}$	$\boxed{.049}$			

[1]The lower chord values at the truss center are outlined in solid boxes. Maximum lower chord deflections which do not occur at center in the King Post truss are underlined.

chord deflection diagrams for each truss. The mean values of deflection at each gage location are shown connected by straight lines that also approximate the elastic curves of the chords. The initial truss data are given in the left-hand diagrams, while those from the trusses with wedges added are given in corresponding positions to the right. Table 2 gives the mean values of deflection shown graphically in Figure 3. The letter-identified locations and placement of the dial gages used to obtain these data are shown in Figure 2.

Table 3 focuses on the single deflection characteristic most commonly treated by engineers and codes, which is the vertical displacement at the midspan of the lower chord in the multipanel trusses and at the point of maximum deflection in the lower chord of King Post trusses. Individual readings are listed in this table along with averages and standard deviations. Truss number 4 in the Howe 2/12 set exhibited progressive deterioration during the deflection procedure and was eliminated in calculating the latter statistics. Allowable deflections, based on the common criteria of span in inches divided by 360 (L/360), are also given for each truss span.

Table 4 summarizes statistical analyses of the deflection readings given in Table 3. The average change in deflection assigned to the wedges (negative if a decrease) has been expressed as a percentage of the average deflection observed in the original trusses without wedges. This figure measures the observed degree of wedge influence. "Student-t" values were calculated using the paired deflection values obtained for each truss, with and without wedges. Tabular *t* values, pertinent to the sample value obtained, are shown in an adjacent column along with their respective probabilities. As shown in Table 4, a significant difference in average deflection change due to addition of the wedge is recognized at the 0.9950-or-above probability level for all truss configurations except for the 2/12 Howe, where a significant difference is recognized at the 0.95 probability level. The King Post truss exhibited a strong response to the wedge, the average deflection difference being

Table 3. — PAIRED VALUES OF PRINCIPAL LOWER CHORD DEFLECTIONS DUE TO DESIGN LIVE LOAD WITH AND WITHOUT WEDGES. THE COMMON DEFLECTION CRITERIA OF SPAN DIVIDED BY 360 IS GIVEN IN INCHES UNDER THE TYPE NAME FOR COMPARISON WITH THE TEST VALUE.

Truss	Deflection location	Truss number	Deflection		Average		Std. deviation	
			No wedge	Wedge	No wedge	Wedge	No wedge	Wedge
Fink 2/12	Center	1	0.805	0.773				
		2	0.827	.815				
		3	0.875	.833				
L		4	0.911	.823				
—— = 1.000 in.		5	1.033	.988				
360		6	0.938	.898				
Span 30 ft.-0 in.					0.898	0.855	0.0827	0.0766
Howe 2/12	Center	1	0.727	.695				
		2	.769	.768				
		3	.736	.730				
L		4	.766	.944[1]				
—— = 0.989		5	.815	.796				
360		6	.729	.722				
Span 29 ft.-8 in.					.755[2]	.742[2]	.0375[2]	.0398[2]
Howe 4/12	Center	1	.252	.234				
		2	.258	.254				
		3	.274	.266				
L		4	.259	.247				
—— = 1.078 in.		5	.291	.282				
360		6	.242	.227				
Span 32 ft.-4 in.					.263	.252	.0174	.0204
King post 4/12	Avg. max.	1	.197	.238				
		2	.180	.227				
		3	.207	.268				
L		4	.186	.249				
—— = 0.656 in.		5	.161	.239				
360		6	.168	.203				
Span 19 ft.-8 in.					.183	.237	.0173	.0218

[1]This number 4 truss developed increasing flexibility as flexing proceeded and then later failed at a low load during strength testing. It is considered most likely that an imperfect lower chord splice was progressively failing during the deflection test processes.

[2]Truss number 4 has been omitted in the calculations.

Table 4. — RESULTS OF PAIRED DIFFERENCE "STUDENT-t" TESTS USING THE DEFLECTION DATA GIVEN IN TABLE 2. THE SAME *t* IS COMPARED WITH THE TABLE *t* AND IF LARGER, THE DIFFERENCE IS SIGNIFICANT AT THE GIVEN PROBABILITY LEVEL.

Truss	Avg. defl. change due to wedge (%)	No. of pairs	Sample t	Table t	Probability level
Fink	−4.8	6	4.23	4.03	0.9950
Howe 2/12	−1.7	5	2.32	2.13	.95
Howe 4/12	−4.2	6	5.33	4.03	.9950
King post	+29.5	6	8.28	6.86	.9995

recognized at a very high probability level. The wedge effect was to increase the deflection of the lower chord through moment sharing with the upper chord, creating a consequent decrease in upper chord deflection as observed in Figure 3.

Table 5 summarizes the data obtained in the subsequent strength testing of the trusses. The load sustained by the truss at the time of failure is given in terms of multiples of design live load because such numbers are most convenient for drawing comparisons and judging performance. The design dead loads were, of course, active at all times during the test and would be added to values obtained from Table 5 to determine total failing loads. Brief remarks concerning failure have been abstracted from the laboratory notes into Table 5 because these contribute much in the interpretation of the results. Since the trusses consisted of only two ingredients, lumber and plates, the failure is primarily identified with one or the other. It should be recognized, however, that the connections depend on interaction between the two ingredients,

Table 5. — ULTIMATE LOAD DATA.[1]

Truss	Truss number	Failing live load factor: No wedge	Failing live load factor: Wedge	Remarks
Fink 2/12	1	2.64		Nail pull, U. C. panel point
	2		3.24	Plate buckle, U. C. panel point
	3	2.78		Nail pull, L. C. panel point
	4		2.46	Same as truss 1
	5	2.63		Same as truss 1
	6		1.84	Lumber failure in bending at edge of heel plate
Average		2.68	2.51	
Howe 2/12	1		2.11	Nail pull, L. C. splice plate
	2		2.32	Same as truss 1
	3		2.36	Nail pull one plate, tension failure other plate, L.C. splice
	4	1.38[2]		Same as truss 1
	5	2.21		Same as truss 1
	6	2.00		Same as truss 1
Average		2.10	2.26	
Howe 4/12	1		1.69	Nail pull, L. C. Splice, lumber split at joint
	2		2.26	Nail pull, vertical web at peak joint
	3	2.88		Nail pull one plate, tension failure other plate, L. C. splice
	4		3.39	Tension failure in plates, L. C. splice
	5	3.17		Nail pull one plate, shear failure other plate, heel joint
	6	1.89		Nail pull, heel joint, fabrication repair required on joint
Average		2.65	2.45	
King post 4/12	1	3.25		Lumber failure, upper chord
	2		4.11	-do-
	3	3.01		-do- plus nail pull at heel plate
	4		4.29	Lumber failure, horizontal shear at heel joint
	5	2.80		Heel plate nail pull
	6		3.90	Lumber failure, upper chord
Average		3.02	4.10	

[1]Each truss carried 20 plf dead load on upper and lower chords plus the upper chord live load required to cause failure. The failing live load factor was obtained by dividing the live load at failure by the 60 plf design live load value. The nails mentioned in the remarks are formed as part of the plate during manufacture. The spans of these trusses were experimental and greater than those permitted by Truss Plate Institute specifications.

[2]This value eliminated from the average per footnote[1], Table 3.

and an event such as a nail pull, for instance, can result from inadequacy in either metal or wood.

Several fabrication problems were encountered that tended to influence performance and thus tended to distort the results. As an example, the lower chord splice connection in the number 1 Howe 4/12 truss contained substantial splitting in one of the members to such a degree that it was noted prior to testing. As another example, the number 6 Howe, 4/12 truss required repair with new plates at a heel joint due to improper initial fabrication. The wood had been punctured by the first plates and further distorted by removal of the first plates which contributed to load failure at this location. In fact, it was necessary to send all of the Howe 4/12 trusses to a different fabricator for repairs that, although well executed, could not help but reduce their performance characteristics.

Another cause of problems in interpreting results lies in the apparent undersize of the splice plates on the Howe 2/12 trusses for the extended experimental spans. In this specimen type, all six of the failures occurred at this joint. A similar design problem appeared in the Fink trusses which showed lack of strength in the connection between the short web member and the upper chord.

An overall examination of the "remarks" column of Table 5 suggests an uncertainty of influence of the wedge in the Fink and Howe truss types but exhibits definite advantage of the wedge in the King Post trusses. The strength data for the King Post were, therefore, subjected to an unpaired "Student-t" analysis yielding a sample t value of 6.3 which compares with a table t of 5.6 required to recognize a significant difference at the 0.9975 probability level.

Discussion

The stiffening effect of the wedge was statistically significant for both the Fink and 4/12 Howe trusses at a high level of confidence. Although the amount of this influence was only about 5 percent, there are cases where such small differences decide between acceptance or rejection of a proposed component or system. The wedge can, therefore, be considered as a possible stiffening aid in trusses when only small amounts are required.

Both the statistical and practical influence of the wedge on deflection in the King Post truss is unquestioned. Although deflection was increased in the lower chord, the action caused was one of sharing moment from the upper chord and reducing deflections in that member. It should also be observed that the higher average deflection after addition of the wedges, 0.237 inches, is still well below the L/360 criterion value of 0.656 inches. There is also the practical aspect of stiffening the lower chord for nailing of ceiling materials and other purposes since the moment-sharing increase of the wedge can also cause the upper chord to participate in the resistance of the lower chord. The advantages of the wedge in low slope King Post trusses may not all be positive because the load sharing effect between chords may cause lower chord deflection beyond allowable limits.

The strength tests are more difficult to interpret with the exception of the King Post type that showed a 36 percent average increase attributed to wedges, which was strongly supported by statistical analysis. With respect to the other trusses, fabrication problems and extension of the spans beyond normal practice caused some difficulty in interpreting the results. Assuming that these effects do not mask the influence of the wedge, it follows that the wedge does not influence the strength of these designs.